

Welded Steel Bridge Girders

Current Practices in Fabricating and
Welding Give Better Appearance,
Higher Strength and Lower Costs

by

OMER W. BLODGETT*

Design Consultant

The Lincoln Electric Company
Cleveland, Ohio

Wider use of welded steel plate girders in bridge construction has developed economies in design, fabrication and erection of such material. With today's high labor costs, it is possible to achieve major reductions in production costs through improved design which trims man-hour-time in both shop and field erection.

Equally noteworthy is the new design freedom experienced with arc welding. Today, it is accepted practice to design and fabricate plate girders with horizontal curves when necessary. Several such bridges or freeway overpasses have been built within the last two years.

A series of four lines of curved welded plate girders with 90-foot spans are a part of the Pasadena-Golden State Freeway's interchange in the Los Angeles area, Figure 1. These have a curve radius of 400 feet. They were fabricated in Kaiser Steel's plant at Montebello.

One of Milwaukee's new expressways has a section of four continuous spans with a total length of 345 feet, in which the two outer girders have a 9 degree horizontal curve and the two inner girders are straight.

Bristol Steel & Iron Works, Bristol, Tennessee, recently fabricated several curved girders for the Southwest Freeway-Inner Loop in Washington, D. C.

Although there are torsional stresses within the curved girder, usually the degree of curvature is not overly high and these additional stresses are offset by the diaphragms connecting the girders. Occasionally, the number of diaphragms has been increased for this reason, and sometimes the allowable stresses have been reduced slightly.

Curved flange plates are flame-cut from plate, laid out by offsets. By cutting both edges at the same time, there is no bowing from any unbalanced shrinkage effect of the flame cutting. The web plates do not have to be preformed, usually being easily pulled into alignment along the centerline of the flanges.

Caution must be used in placing attaching plates for the diaphragms to the webs and flanges. The proper angle for these plates may vary along the length of the girder. Shear attachments are added mainly to accomplish composite action between the concrete deck and steel girder, and thereby provide added torsional rigidity. During erection, a pair of curved girders is attached together by means of the diaphragms and then hoisted into position as a unit.

In addition to curved girders, a number of recently completed bridges of unusual and unique design provide standing examples of this elimination of restrictions on the engineer's creative design ideas. Among them are the Buffalo Bayou Bridge on U.S. 90A in Houston, Texas; Nisqually River Bridge in Mt. Rainier National Park in Washington; the Whiskey Creek Bridge in California; and the

* Member AWS Conference Committee on Welded Bridges.

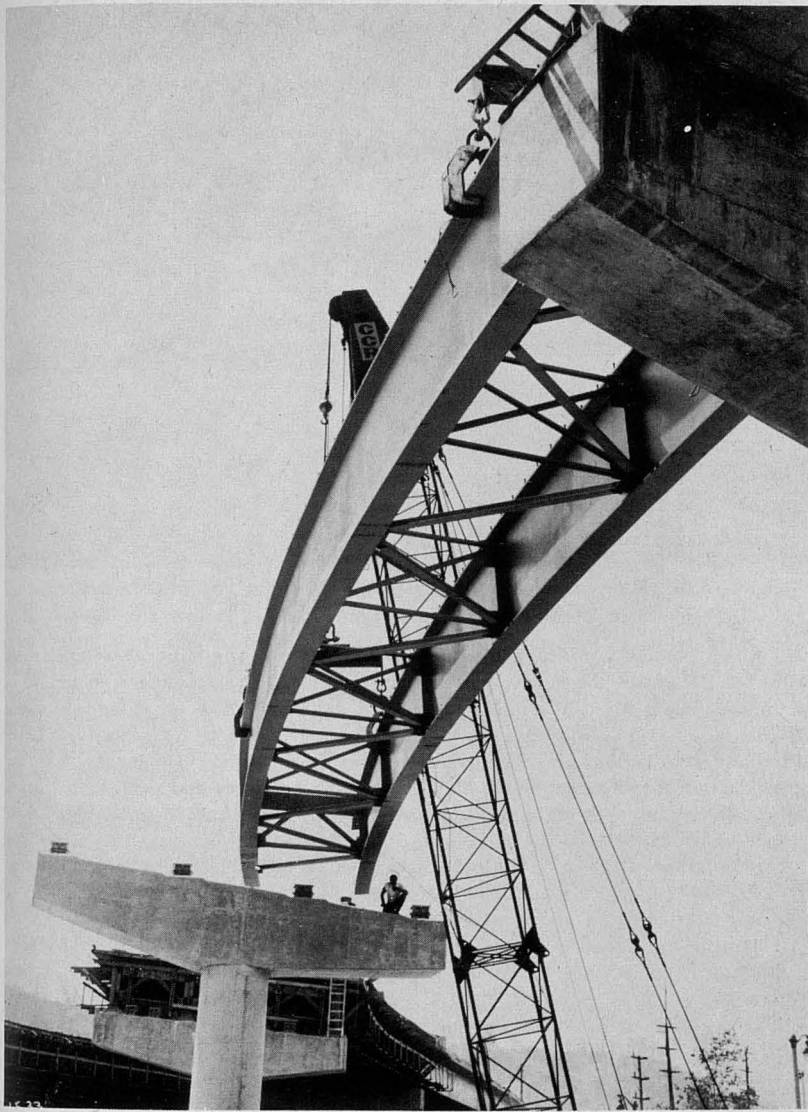


Figure 1. - Horizontally curved girders in Pasadena-Golden State Freeway's interchange in Los Angeles, California.

1/4 mile long Calcasieu River and Ship Channel Bridge in the Lake Charles area of Louisiana.

BUFFALO BAYOU BRIDGE

Two entirely separate units on U.S. 90A in Houston, Texas constitute the Buffalo Bayou Bridge, Figure 2. Each bridge has four lines of girders based on 8 foot cen-

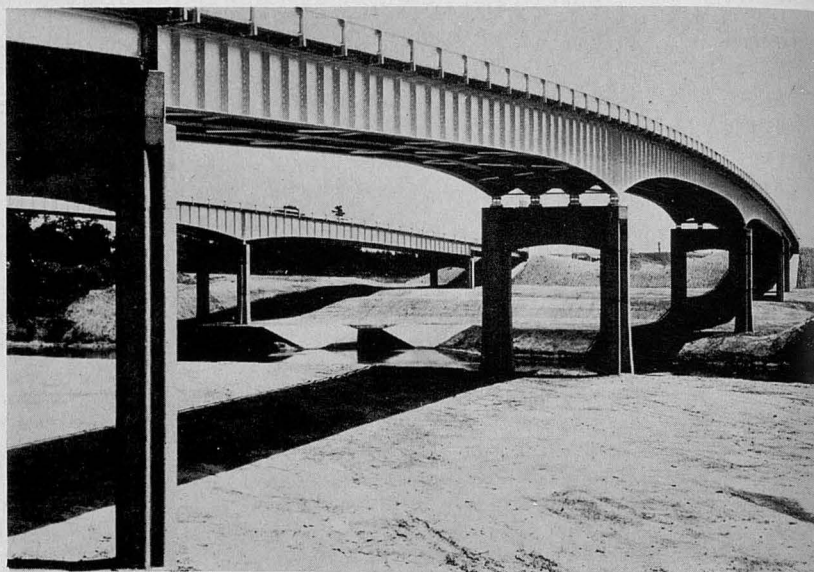


Figure 2. - The Buffalo Bayou bridges on U.S. 90A in Houston Texas.

ters which continuously span 198, 270 and 198 feet. The flange has a constant width of 24 inches the entire length of the structure. The web at the centerline of the main span is a $1\frac{1}{2}$ inch thick plate with a depth of 6 ft. 6 in. Flange thickness at this point is $1\frac{7}{8}$ inches thick. Over the piers, the web increases to a 12 ft. depth and $\frac{9}{16}$ in. thickness. Flange thickness at this point is $1\frac{5}{8}$ inches. Welding specifications for the continuous fillet welds joining flanges and webs called for a $\frac{5}{16}$ inch leg size for flanges up to $1\frac{1}{4}$ inch and $\frac{3}{8}$ inch leg size for flanges above $1\frac{1}{4}$ inches thick.

Longitudinal stiffeners on the inside of the girders allowed the web depth-to-thickness ratio to be increased to 254. Transverse stiffeners were used on both sides of the web.

Where a transition in flange thickness occurred, the thicker plate was beveled with a 3 in 12 in. slope.

Design of the deck structure used a light weight concrete to reduce dead weight and resulted in an 8% reduction in steel requirements. Shear attachments were used in the positive moment regions to produce a more rigid structure and assist in damping out vibrations. The composite action of the concrete and steel was not considered in design calculations, however, due to the uncertainty of the physical properties of this light weight concrete.

NISQUALLY RIVER BRIDGE

The Nisqually River Bridge, Figure 3, 375 feet long, spans 300 feet and is cantilevered out on one end another 75 feet. Girder webs are $\frac{3}{4}$ inch thick plate and have a constant depth of 16 feet. The flanges, varying from 27×1 inch up to 29×4 inches are joined to the web by a fillet weld whose size corresponds to the minimum value set by the AWS Bridge Specifications. The transition in flange thickness and width was made with a slope not exceeding 1 inch in $2\frac{1}{2}$ inches.

The girders were shop fabricated into upper and lower half sections complete with stiffeners. Longitudinal stiffeners were used on the inside of the girder and transverse intermediate stiffeners on both sides. After shipment to the erection site, these half sections were field spliced using semi-automatic submerged arc welding equipment. This longitudinal butt weld splice occurred along the centerline of the web, and was completed with the girders still at ground level. After completing this butt weld, the splices in the transverse stiffeners were welded and the girders then hoisted into position.

WHISKEY CREEK BRIDGE

California's Whiskey Creek Bridge, also has a constant girder web depth. This 12 foot dimension, however, was achieved by the design integration of A-373, A-242, and T-1 steel. The physical properties of these three grades of structural steel were used to advantage by selecting A-373 for the lower stressed areas, A-242 for the intermediate stressed areas, and low alloy-high tensile T-1 in the bridge's highest stressed points. Except for the 90 foot central section of the main span where 2 inch flange plates were used, girder flanges are a constant 1 3/4 inches thick.

Final design placed longitudinal stiffeners on the outside of the girder and transverse intermediate stiffeners on the inside. Transverse stiffeners were also placed on the outside adjacent to interior stiffeners where diaphragms were attached. Resulting ratio of web depth to thickness was increased to 270 for A-373 steel, 220 for A-242 steel, and 164 for T-1 steel. In general the same type of steel was used in the flange and web of the fabricated girders. During the design stage, however, consideration was given to the use of a lower strength web with a higher strength flange. Research efforts at the University of Texas has proved this "hybrid" design approach will develop a high carrying capacity. In the Texas efforts, girders using T-1 flanges with A-36 webs were successfully tested with loads which stressed the web at a working load above the yield point. The state of Iowa has been doing similar investigation.

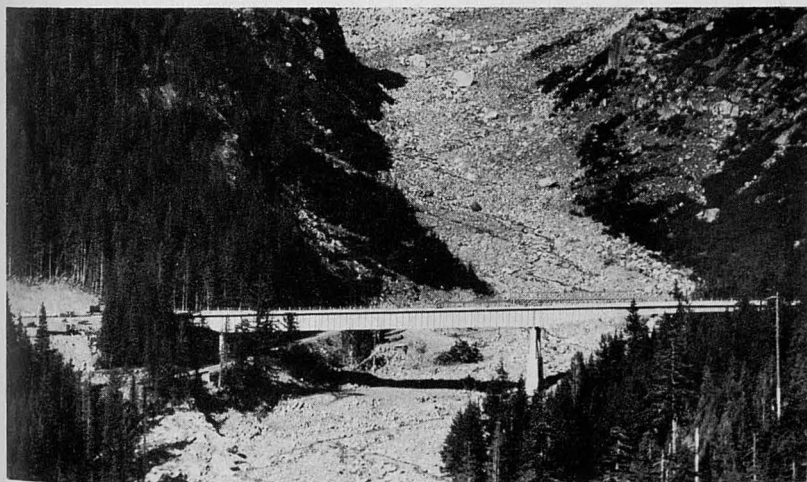


Figure 3. — Nisqually River Bridge in Mt. Rainier National Park in Washington.

The Whiskey Creek Bridge design limited deflection to AASHTO Specifications by reducing the bending stress in the T-1 flanges in the region of negative moment near the support from usual 45,000 psi to 39,700 psi. The 30 in. x 2 in. T-1 flange in the central section, with an allowable of 39,700 psi, would have had to be increased to 4 1/2 in. thick if A-242 steel were used, with an allowable of 22,000 psi. Use of A-373 steel with an allowable of 18,000 psi, would have called for 5 3/4 in. thick girder flanges. In addition, the elimination of high strength steel would have resulted in a design involving deeper webs which would have varied in depth. Naturally, much of the structure was shop fabricated using semi and full mechanized welding techniques wherever possible. Field welding specifications called for the use of low hydrogen electrode.

CALCASIEU RIVER BRIDGE

This bridge, Figure 4, erected in the Lake Charles area of Louisiana has three continuous spans of 225, 450 and 225 feet. The total length of the entire structure including approach spans is 6,780 feet and comprises a total of 47 spans. Approach girders range from 75 to 180 feet in length.

The final design used low-alloy high-strength A-441 steel in girder flanges and webs. The 14 foot x 5/8 in. web in the central section of the main span increases to 20 feet 10 in. x 1 in. over the pier support. Flanges vary from 24 in. x 1 in. up to 36 in. x 4 in. at the center.

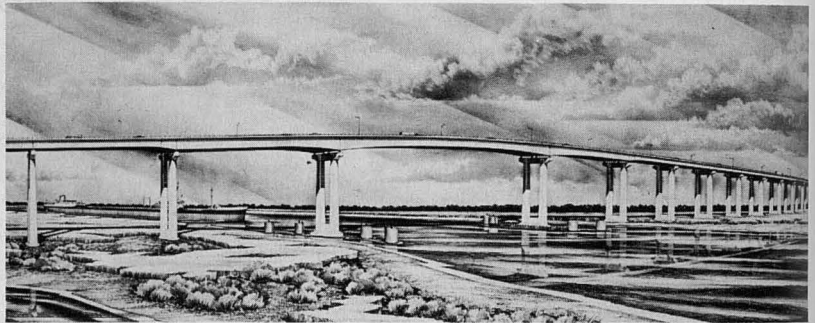


Figure 4. - Calcasieu River Bridge, Lake Charles, Louisiana.

Transition in flange thickness was accomplished with a slope of 1 to 6 inches and called for beveling the thicker plate. Transition in flange plate width has the wider plate tapered for a distance of 6 feet back from the joint.

Fillet welds joining the flange to web correspond to the minimum value set by the AWS Bridge Specifications. Longitudinal stiffeners were placed on the inside of the girders, and transverse intermediate stiffeners on both sides.

A relative comparison of the girders for these bridges is seen in Figure 5. Since the girders are positioned relative to completion date with the latest on the extreme right, the trend in girder design is obvious. As the bridge engineers gain experience and confidence in the fabricated girders, their design called for increased girder depth and longer spans.

FLANGE-TO-WEB WELDS

These welds hold the flanges to the web of the plate girder. They are located in areas of bending stresses and must transfer longitudinal shear forces between

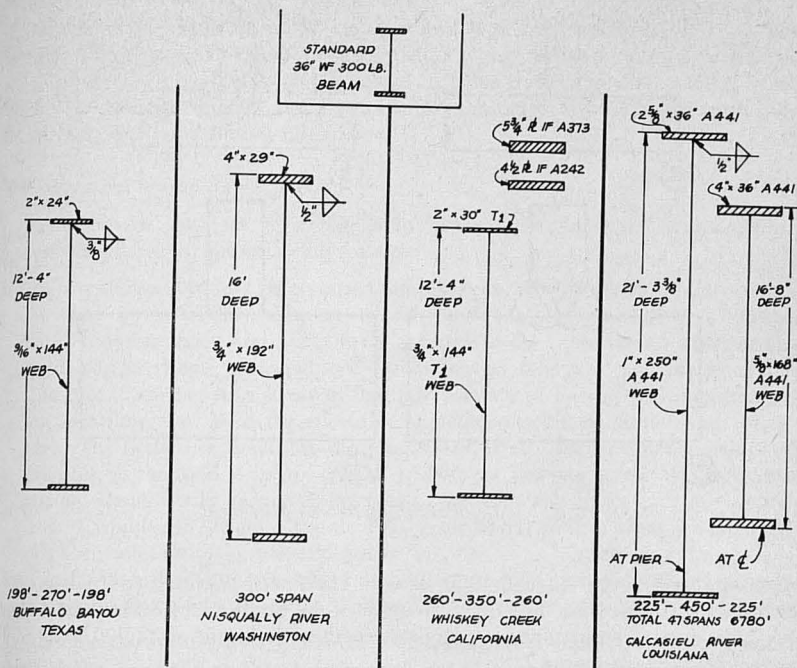


Figure 5. — Comparison of the girder design of four well known plate girder bridges erected in the United States.

the flanges and the web. Some restraining action may develop with the thicker flange plates, but any resulting transverse residual stress should not reduce the load carrying capacity of the weld. This being parallel loading, the actual contour or shape of the fillet weld is not as critical as long as the minimum throat dimension is maintained.

Shop welding techniques today usually used submerged arc automatic welding equipment to make these welds. For the usual thickness of web plate, the two fillet welds penetrate deeply within the web and intersect (see *b* in Figure 6), giving complete fusion even though simple fillet welds are called for (as in *a*). Some states recognize this penetration and are now detailing this weld with complete fusion. This proves no problem on the normal web thickness. In the future, however, if the same detail is shown on much thicker web plates, the fabricator will have to use a double bevel edge preparation to obtain the intersection (*c*), even though detail (*d*) is sufficient.

It should not be necessary to detail groove welds for this joint from a design standpoint. Selection of a groove T-joint design should be based on a cost comparison with fillet welds. The grooved T-joint requires about 1/2 the amount of fusion metal compared with fillet welds (assuming full strength welds). However, the grooved joint has the extra cost of preparing the double bevel. In respect to the physical performance of either the fillet or the grooved T-joint design, A. Newmann, working at Cambridge University, tested these welds under fatigue bending from 0 to tension, *K* equals 0, at 2,000,000 cycles. The results of his test reported in an article entitled "Discussion at the Symposium on Fatigue of Welded Structures"

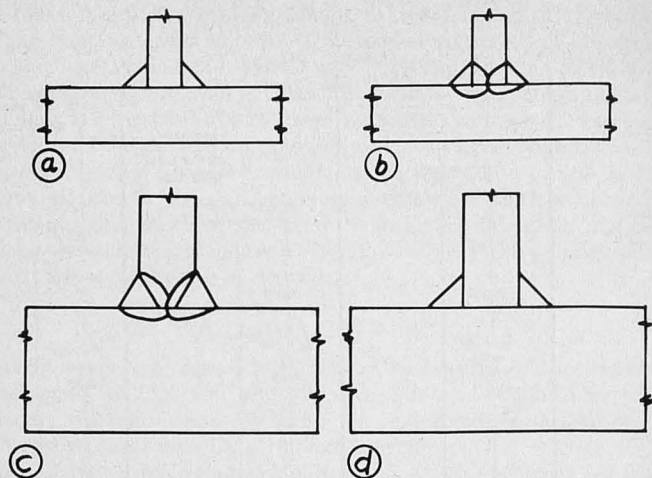


Figure 6. - Typical flange to web fillet welds.

which appeared in the August, 1960 issue of The British Welding Journal, no difference was indicated for the fatigue strength of the beam using either joint design with both types demonstrating a fatigue strength in the beam of 22,000 to 24,000 psi (bending stress), Figure 7.

FILLET WELDS

Minimum Size

From a design standpoint, these welds may be quite small. Their actual size is usually established by the minimum allowable leg size for the thickness of the flange plate. Table 1 lists the minimum size of fillets for various plate thicknesses as established by AWS Specifications. Leg size is increased to take care of the faster cooling rate and greater restraint that exists in thicker plates.

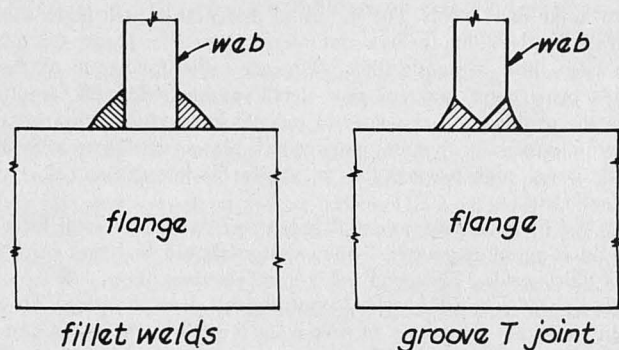


Figure 7. - Fatigue bending tests on these two types of welds joining girder web to flange demonstrated no difference in fatigue strength.

On thicker plates, with multiple pass welds, it is desirable to get as much heat input into the first pass as possible. This means higher welding currents and slower welding speeds. Low-hydrogen electrodes are better for manual welding in this work. The low-hydrogen characteristics of a submerged arc welding deposit gives this welding method a similar advantage.

Determination of Combined Stress

The combined stresses in a fillet weld between the girder web and flanges is seldom considered for the following reasons:

1. The maximum bending stress for a simply supported girder does not occur at the same region as the maximum shear force. For a continuous girder, however, the negative moment and shear force are high in the same region near the support, and perhaps the combined forces in this fillet weld should be checked.
2. The maximum bending stress in the outer surface of flange is always designed for something less than the allowable (Bridge code for example equals 18,000 psi). The weld lies inside of the flange and is stressed at a lower value. For example, if the weld is in an area of 15,000 psi bending stress, this additional normal stress would reduce, theoretically, the allowable shear force for the weld from f equals 8800w to f equals 7070w, or about 80% of what it would be if just horizontal shear were considered.
3. Usually these welds must be larger than design requirements because of the minimum weld size specifications listed above.

Nevertheless, should it be desirable to determine the combined stresses, it can be theoretically shown that the axial normal stress from the bending, applied to the fillet weld, would increase the maximum shear stress applied to the throat. For a given applied normal stress (σ), the resulting maximum value for the allowable force (f) which may be applied to the fillet weld of a given leg size (w) under parallel loading is expressed by the formula:

$$f = w \sqrt{8800^2 - \frac{\sigma^2}{8}}$$

This formulation still permits the maximum shear stress resulting from the combined shear stresses to be held within the allowable of equals 12,400 psi (Bridge allowable).

Allowable Fatigue Strength

Table II contains the formulas for establishing the allowable shear force that may be applied to fillet welds under various conditions of fatigue loading.

PLATE GIRDER WEBS

AASHTO Specifications require that the thickness-to-depth ratio of girder webs be not less than the values indicated in Table III.

This ratio of web thickness to clear depth is based upon predictions of the plate buckling theory, the web plate being subjected to shear throughout its depth and compressive bending stresses over a portion of its depth. The plate buckling theory assumes the portion of the web to be an isolated plate; however, in the plate girder, the web is part of a built-up member. When the critical buckling stress in the web is reached, the girder does not collapse. The flange plates carry all the bending

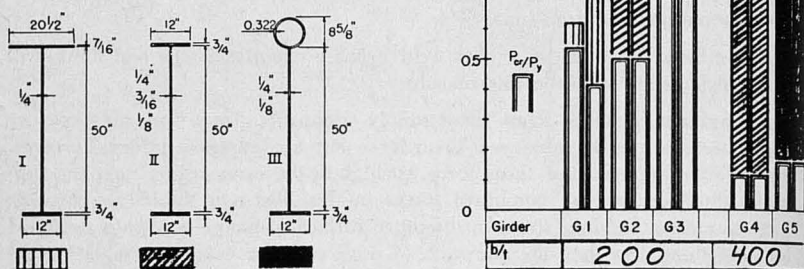


Figure 8. — Results of the Plate Girder Research Test at Lehigh University.

moment, the buckled web serves as a tension diagonal and the transverse stiffeners become the vertical compression members, in effect making the girder act as a truss.

Research at Lehigh University tested, among other things, the effect of the web thickness on the ultimate carrying capacity of the girder, Figure 8. It was found that the ultimate load carrying ability of the girder, expressed as the ratio of the ultimate load to load causing yield stress, was directly proportional to the restraint offered by the compressive flange. The more torsionally flexible flange (wide and thin) resulted in the lower strengths, and the more torsionally rigid flange (tubular) in higher strengths.

Different web slenderness ratio produced little effect on the ultimate load-carrying capacity of the girder for the same compression flange.

Although the tubular type of compression flange was used to obtain a torsionally rigid flange—and it is not recommended for actual practice—it should be remembered that the concrete floor slab directly on top of the compression flange offers a similarly high torsional restraint, as well as good lateral bracing.

Some bridges have longitudinal stiffeners on the inside of the girders, others on the outside. If the longitudinal stiffeners are on the inside, along with the transverse stiffeners, it leaves the outside of the girder smooth. This, of course, means that the longitudinal stiffener must be cut into short lengths and then inserted between the transverse stiffeners. This results in increased welding time and production costs.

Some states—New York and Kansas to mention two—have used longitudinal stiffeners on the outside and transverse on the inside. As mentioned, this method saves on fabricating time and also allows the use of automatic welding techniques to join the stiffeners to the girder web thereby substantially increasing welding speed.

COVER PLATE ENDS

Many methods have been suggested for the terminations of cover plates. The existence of at least four conditions which affect this termination complicates the

situation and makes it impossible to recommend one specific cover plate end which will best meet all conditions.

First, the tensile forces, assumed to be uniformly distributed across the width of the cover plate should be transferred simply and directly into the corresponding flange of the rolled beam without causing any stress concentration in the beam flange. In general, a large transverse fillet weld across the end of the cover plate does this in the simplest manner.

Second, there must be a very gradual change in section of the beam at the termination of the cover plate to develop a similar gradual change in bending stress of the beam. Any abrupt change in beam section will reduce the fatigue strength of the beam. This would tend to favor a gradual tapered width at the end of the cover plate.

Third, some consideration should be given to the termination of the cover plate in the narrow zone of the flange in direct line of the web of the beam, since this is a rigid portion with little chance for localized yielding to prevent the build-up of possible high stress concentration.

Fourth, the selected joint should be economically practical to make and answer functional requirements. For example:—

1. Continuous welds may be needed to provide a positive seal and prevent moisture from entering underneath the plate and causing connection deterioration.
2. Minimum appearance standards may eliminate some joint designs.

Early fatigue testing at the University of Illinois on rolled beams with cover plates (Bulletin No. 377, Jan. 1948) indicated that:

1. In general, continuous fillet welds were better than intermittent fillet welds for joining cover plates to the beam flange.
2. On wide, thin cover plates extending out beyond the width of the beam flange and connected with longitudinal $\frac{3}{16}$ in. continuous fillet welds; adding a $\frac{3}{16}$ in. fillet weld across the end of the cover plate produced a slight increase in fatigue strength (from 8900 psi to 9300 psi at 2,000,000 cycles). Omitting the welds for a distance at each corner of the cover plate increased this value up to 10,000 psi, see Figure 9.
3. For the narrow, thick cover plates lying within the width of the beam flange, increased fillet weld size across the end of the cover plate produced a gradual increase in fatigue strength; a $\frac{5}{16}$ in. fillet weld had a strength of 9400 psi at 2,000,000 cycles, a $\frac{3}{8}$ in. fillet weld 11,000 psi, and a $\frac{3}{8}$ in. x 1 in. fillet weld went up to 12,600 psi. This particular size of cover plate was not tested with the transverse fillet weld omitted, see Figure 10. These values compare well with more recent work at Illinois by W. H. Munse and J. E. Stallmeyer.

The latest work reported at the University of Florida on 18 in. WF 70 lb. beams with 5 in. x $\frac{5}{8}$ in. cover plates showed that the beam flange within the cover plated region was stressed lower when a $\frac{5}{8}$ in. fillet weld was placed across the end of the cover plate as compared to that with no transverse weld. The transverse weld also produced a more uniform distribution of stress across the cover plate as well as the beam flange and allowed the plate to pick up its share of the beam force in a shorter distance. However, all of these factors occur within the cover plated region of greater section modulus and lower bending stress, so this is not very serious.

Cover Plates Extending Beyond Width of Beam Flange

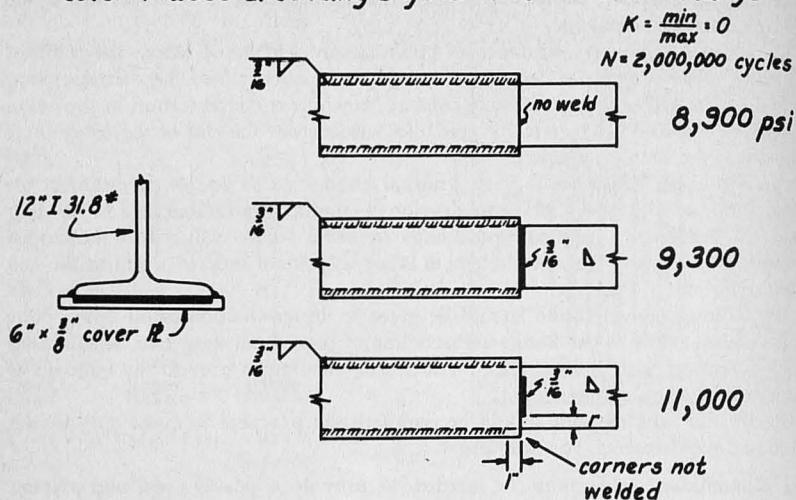
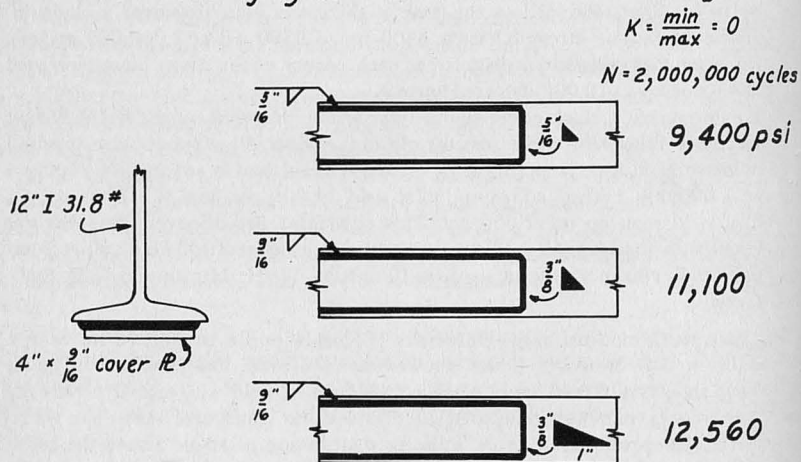


Figure 9. - Comparison of fatigue strength of cover plates extending beyond width of beam flange with variations in weld specifications.

Cover Plates Lying Within Width of Beam Flange



no tests made with the transverse fillet weld left off

Figure 10. - Comparison of fatigue strength of cover plates within width of beam flange with variations in weld specifications.

What is more important is the effect the transverse weld and shape of the end of the cover plate has upon the stress in the beam flange adjacent to the point where the cover plate is attached. This is the region of lower section modulus and higher bending stress and is much more critical than any region within the cover plate.

The drawing, Figure 11, illustrates variation in terminations of cover plates. The data summarizes recent tests on the fatigue strength of beams with partial cover plates, conducted at the University of Illinois. Although the common method of terminating the cover plate directly across the flange with a transverse fillet weld is satisfactory and acceptable by the AWS Bridge Specifications, this data would seem to indicate that tapering the end of the cover plate and eliminating transverse welds across the end slightly increases the fatigue strength. It should be noted that a small 1/4 in. fillet weld was used when welded across the end of the 1/2 in. thick cover plate. The results may be different if a larger transverse weld had been used. Most states require continuous welds on cover plates and across their ends thereby eliminating the selection to termination types *a* or *b*. Since the data indicates that tapering has little effect, final selection between *a* or *b* would have to be made on some other factor such as appearance, or lower dead weight.


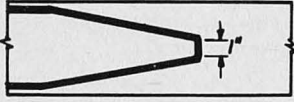
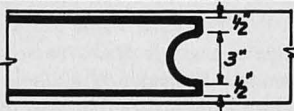

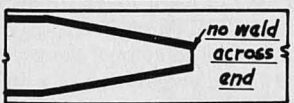
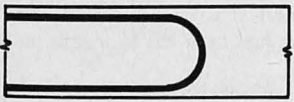
Type of termination of cover plate	K=0 N=100,000 ~	K=0 N=2,000,000 ~
	26,500 psi	11,300 psi
	33,000	11,500
	30,700	14,500
	34,700	12,500
	36,500	13,700
	29,000	11,700

Figure 11. - Suggested methods for terminating cover plates.

In summary, it would appear that the short section of the transverse weld across the end of the cover plate directly over the web of the beam is restrained and when tested under severe fatigue loading may reduce the fatigue strength of the connection unless it is made large. A large transverse fillet weld, especially in this central section would more uniformly transfer this force through the surface of the beam flange into the end of the cover plate. See Figure 12.

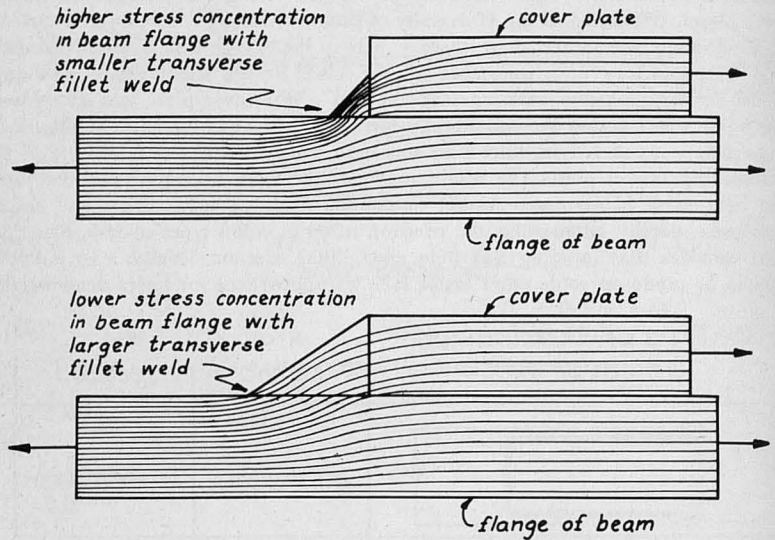


Figure 12. - When transverse fillet welds at termination of cover plate are called for, make them large.

FLANGE BUTT JOINTS

In nearly all welded plate girders, the flange is a single plate. These plates are stepped down as less area is required. A smooth transition is made between the two, either by reducing the thickness or the width of the larger flange to correspond to that of the smaller.

When this transition is made in thickness, the end of the larger flange is beveled by a flame-cutting torch. There is a practical limit on the angle of bevel, but this slope, according to AWS Bridge Specifications, should not be greater than 1 in 2 1/2 inches or an angle of 23 degrees. On the Calcasieu River Bridge this slope is decreased to about 1 inch in 6 inches, or an angle of about 9 1/2 degrees. Transitions also can be made by varying the surface contour of the groove welds.

The usual method of flame cutting a bevel in the preparation of a welded joint is to cut down through the surface of the plate at the proper angle. Because of the wide angle needed for this transition in thickness, it is often better to flame-cut back from the edge of the plate after the flange plate has been cut to length, as illustrated in Figure 13.

When the transition is made in width, the end of the wider flange is cut back at an angle, again with the flame-cutting torch. There is no problem in cutting in this manner, and any slope may be used; many times 1 in 12, but usually a maximum slope of 1 in 4. Often this taper may extend back for several feet. Generally, it

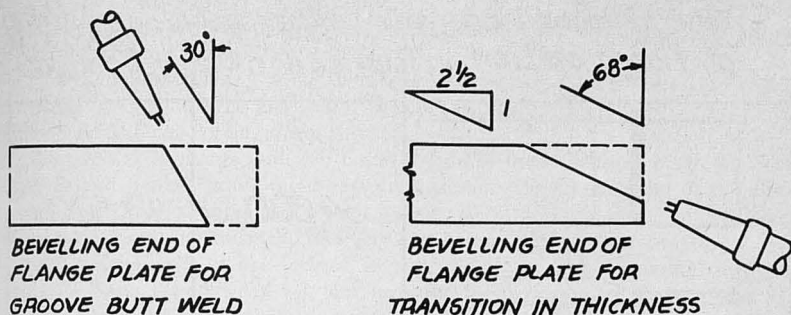


Figure 13. — Methods of beveling girder flanges for butt welding or transition in flange depth.

is felt that the straight-line transition in width is sufficient, and in the case of fatigue loading the allowable fatigue values butt welds in tension or compression are used. If a curve, tangent to the edge of the narrow flange at the point of termination is used, it may be assumed that the flanges have equal widths, and equal thicknesses, with the weld reinforcement removed, the butt weld may be assigned the same allowable stress as the flange plate, under any condition of fatigue loading.

Studies at the University of Illinois have indicated a slight advantage in making a transition in width rather than in thickness, Figure 14. This advantage undoubtedly would be greater if the transition in width were made more gradual; however, both methods are sound and acceptable. Fatigue values for these transitions are found in Figure 15.

In butt welding the ends of flange plates, some thought should be given to the proper type of joint. J and U joints require the least amount of weld metal; however, in general, they are prepared by some type of machining, planing or milling, and this is not practical in most fabricating shops. This limits the preparation to flame beveling, giving a V joint.

In the V joint, less weld metal is necessary, as the included angle is decreased; however, as this angle decreases it is necessary to increase the root opening to get the manual electrode down into the joint and produce a sound weld at the root of the joint. Obviously, the one tends to offset the other slightly in respect to the amount of weld metal needed. On thicker plates, the smaller included angle joint, with its larger root opening, requires the least amount of weld metal.

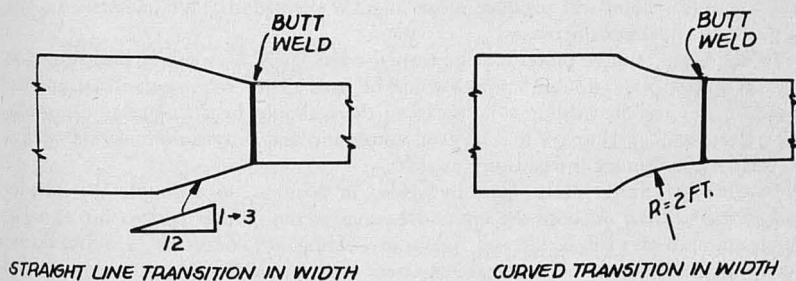


Figure 14. — Methods of tapering flanges for transition in flange width.

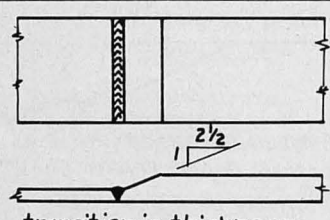
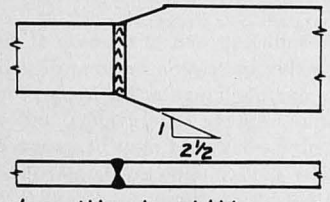
Type of transition of flange section	$K=0$ $N=100,000 \sim$	$K=0$ $N=2,000,000 \sim$
 transition in thickness	34,600 psi	18,500 psi
 transition in width	34,900	19,500

Figure 15. - Recent University of Illinois studies indicate a slight fatigue strength advantage to making a flange transition in width rather than depth.

If a backing strap is used, any amount of root opening can be tolerated, within reason, and, of course, all of the welding must be done on the same side, in other words, a single V joint. If a backing strap is not employed, then this root opening must be held to about 1/8 in. This enables the root pass to bridge the gap and not fall through. The welding may be done all on one side—single V—or it may be done on both sides—double V. In either case, the joint is “back-gouged” from the opposite side of the first pass to the root of the first pass before depositing the weld on this “second” side. This will insure sound metal throughout the entire joint.

Single V joints may be acceptable if the plates are not too thick; for thicker plates, double V joints are preferred since they require less weld metal. Remember that a single V joint will produce more angular distortion. This increases rapidly as the flange thickness increases.

In the shop, flange plates can be turned over easily as welding progresses, so that on thicker plates double V joints would be used. They require the least amount of weld metal and the welding is balanced so there should be no angular distortion. On wider plates, perhaps 2 to 3 ft., semi-automatic and fully automatic submerged arc welding equipment is frequently used.

In the field, flange plates must be welded in position, so a double V joint with half of the welding on both the top and bottom of the joint is best as far as minimum amount of weld metal and balanced welding are concerned. Unfortunately, however, this requires a considerable amount of overhead welding. For this reason, the AWS Bridge Prequalified Joints allow the double V to be prepared so that a maximum of three-fourths of the joint is on top and the rest on the bottom. The

slight increase in weld metal required as compared to the equal double V is more than offset by the reduction in overhead welding.

Most butt welds joining webs are single V. However, thicker webs may call for a double V to reduce the amount of welding and permit the balancing of welding on both sides to control angular distortion.

Shop splices in flange and web plates should be made before the girder is fitted together and welded, providing the resulting sections are not too long or too heavy to handle. These shop splices do not have to be in a single plate, but are placed where they are most convenient, or where a transition in section is desired.

Field splices usually are located on a single plane. The staggering of the butt welds of flanges and webs will not improve the performance of the girder. It is much easier to prepare the joints and maintain proper fit-up by flame-cutting and beveling when all are located in the same plane. The flanges may be fillet welded to the web all the way to the very end of the girder. This provides better support when the flanges are clamped together for temporary support during erection.

PROPER FIT-UP

Good fit-up is essential to the development of efficient welding procedures. This means proper alignment and correct root opening. Placement of flange and web butt splices in the same plane greatly increases the possibility of achieving correct root opening when the girder is pulled into alignment. Figure 16 illustrates a misaligned double V butt joint in a girder flange at the point of transition. Note the offset of the joint preparation makes it difficult to reach the root of the joint and deposit a sound weld throughout the entire joint. The flange joints should be checked for alignment throughout their entire length before welding. This illustrated condition can exist at the extremities of the flanges even though perfect alignment exists in the web area. Accidental tilt of the flanges during fabrication, mishandling during movement to the job site or even a difference in warpage of the two flanges can cause this condition. The warpage problem increases with the size of fillet weld and decreases as the flange thickness increases.

Various methods exist for correcting this condition. Figure 17 illustrates one

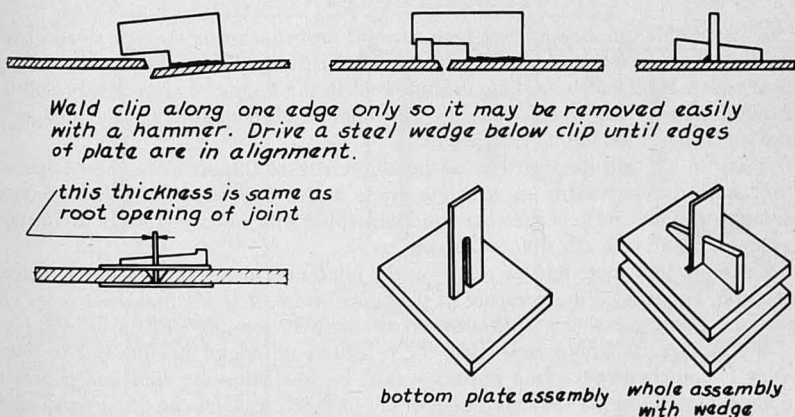


Figure 17. - Small clip angles and wedges provide an economical method for bringing two plates into alignment. Welding clip on one side only makes it easy to remove by blow with a hammer.

such method. When the plates are not too thick, a small clip can be welded to the edge of one plate. Driving a steel wedge between this clip and the other plate will bring both edges into alignment. A note of caution to the welding operator when this technique is used. Welding on just one side of the clips greatly simplifies their removal. Figure 18 illustrates still another method which receives wide use when problems develop in respect to misaligned thicker flanges. Here a heavy bar or strong back is pulled up against the misaligned plates by driving steel wedges between the bar and attached yokes (top view). An alternate method involves the use of bolts welded to the misaligned plate and then drawing the misaligned plate up against the strong back with bolts.

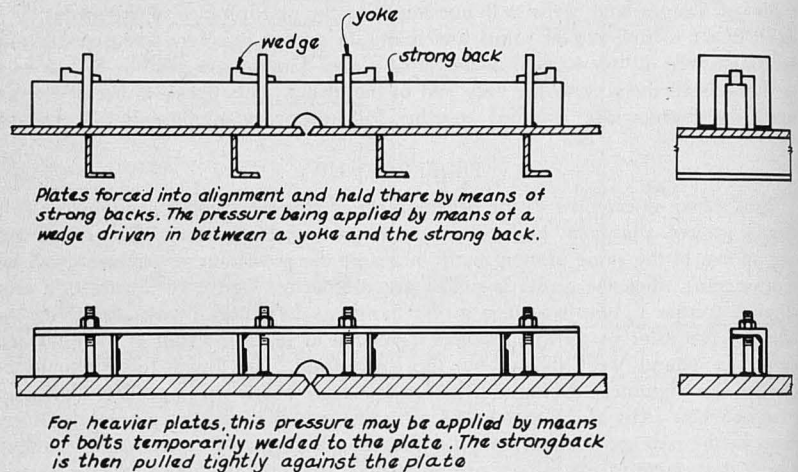


Figure 18. - Wedges and bolts are used to pull plates into alignment on "Strong-backs."

COPED HOLES IN WEB AT SPLICE

Considerable questioning has been directed toward whether the web should have coped holes to aid in making the butt weld in the flange for the field splicers. Since these coped holes aid in making the butt weld in the flange, it is necessary that the disadvantage of the coped holes be carefully weighed against the advantages of making a sound weld in the flange butt.

Tests on 12 inch deep girders at the University of Illinois have shown that the field splice having welds in a single plane and using coped holes has a fatigue strength of about 84% of the corresponding splice with no coped holes at 100,000 cycles, or about 90% at 2,000,000 cycles.

Knowing that these figures represent the maximum reduction possible in fatigue strength, because of the presence of the coped holes, it is felt that these holes will do more good than harm since they insure the best possible weld in the butt joint of the flanges. It would seem that the reduction in fatigue strength due to coped holes on much deeper plate girders would be less, since the reduction in section modulus ascribable to the coped hole would be much less. Of course, a notch effect of the coped hole would still be present. If necessary, this hole can be filled by welding after the flange butt weld was completed.

ALLOWABLE FATIGUE STRENGTHS

Grooved welds in butt joints of equal thickness, if the reinforcement is finished smooth with the surface, may be allowed the same fatigue strength under any type of fatigue loading as the base metal. For plates of unequal thickness, where the transition slope is not greater than 1 in 2 1/2, the formulas found in Table IV may be used. These assume no abrupt change in section, greater than the 1 in 2 1/2 slope.

WELDING OF STIFFENERS

AWS Bridge Specifications in Paragraph 225(c) do not allow the welding of stiffeners to tension flanges if the stress exceeds that of the allowable at the termination of a fillet welded cover plate.

Some engineers have felt this reduction in fatigue strength is due to the transverse fillet welds; however, it is caused by the abrupt change in section due to the attachment. It is believed these plates would have failed at about the same value and location if they had been machined out of solid plate without any welding. This same problem exists in the machining of stepped shafts used in large high-speed turbines and similar equipment.

Figure 20 illustrates the effect of welding transverse stiffeners to tension flanges. Reference again may be made to "Flexural Strength of Steel Beams," Bulletin 377, University of Illinois, 1948. Tests were made from tension to zero tension in bending (k equals 0) and at 2,000,000 cycles.

Eliminating the weld between the stiffener and the tension flange increased the fatigue strength of the beam. Leaving the weld off the lower quarter portion of the web in the tension region in addition gave a further increase in fatigue strength.

EFFECT OF WELDED INTERMEDIATE STIFFENER ON TENSION FLANGE

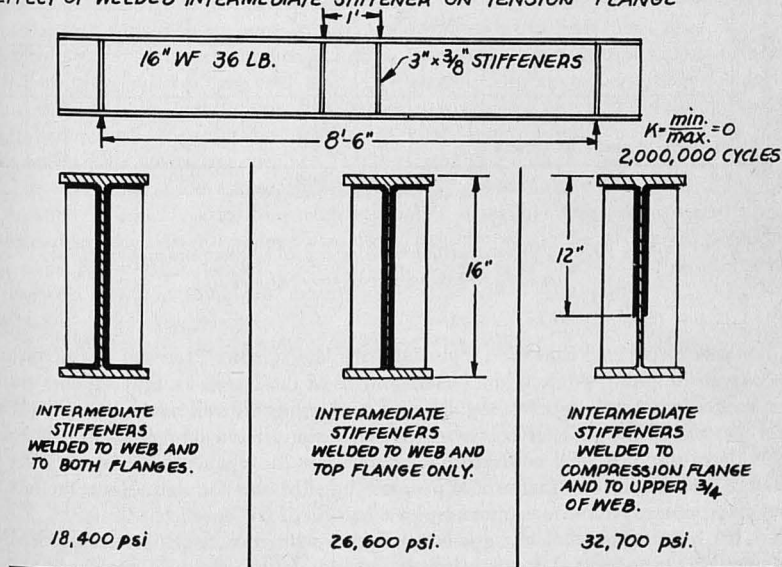


Figure 20. - Effect on welded transverse stiffeners on tension flange.

Later tests at the University of Illinois on the welding of stiffeners to girders took into consideration not only the bending stress of the flange, but also the resulting principal tensile stress in the web at critical locations, such as the termination of the connecting fillet weld of the stiffener. You find this illustrated in Figure 21.

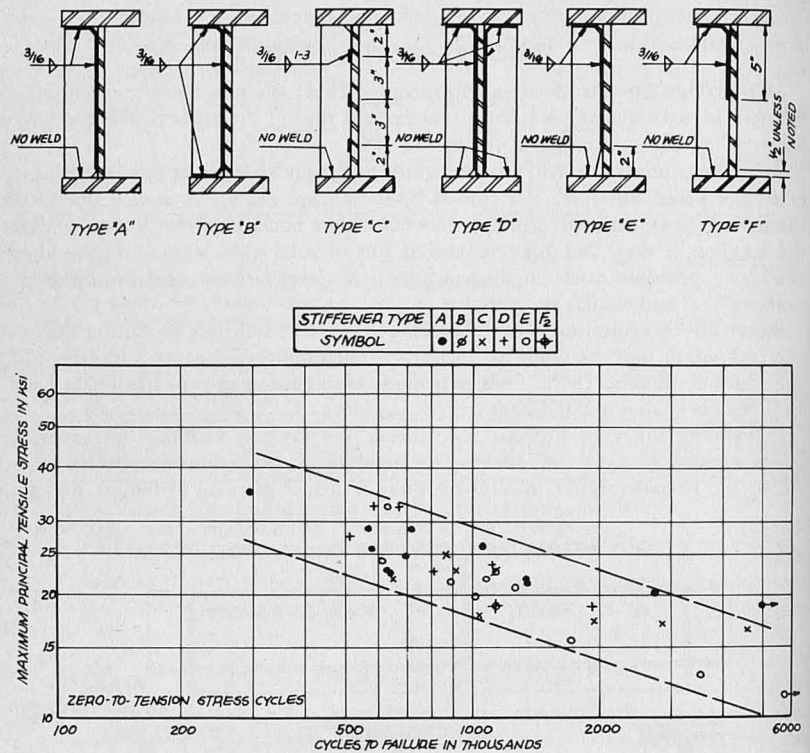
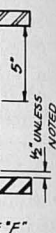


Figure 21. - Types of stiffeners and a plot of maximum principal tensile stress values versus fatigue life.

It was discovered that the fatigue failure in the stiffener area did not necessarily occur at the point of maximum bending stress of the beam. Fatigue failures started at the lower termination of the fillet weld connecting the stiffener to the web. When the bottom of the stiffener was also welded to the tension flange, failure started at the toe of the fillet weld connecting the stiffener to the flange of the beam. After the flange had failed, the crack would progress upward into the web. Here, the failures usually occurred in the maximum moment section of the beam.

This test indicated fairly good correlation when the results were considered in terms of the principal tensile stresses (including the effect of shear) rather than simply the bending stress. The angle of the fatigue failure in the web generally was found to be about 20% less than the computed angle of the principal stress.

girders
to the re-
me termi-
l in Fig.



AASHTO specs, Paragraph 2.10.32, state that transverse intermediate stiffeners shall fit sufficiently tight to exclude water after painting. Some inspectors interpret a tight fit to be one in which the stiffeners must be forced into position. Many fabricators feel this is an unnecessary deterrent since it takes extra time to force the edges of the flanges apart to allow the stiffeners to be inserted. There are, in general, two methods of fitting these stiffeners to the plate girder: -

1. a. Use a stiffener that does not fit too tight.
- b. Push the stiffener tightly against the tension flange.
- c. Weld it to the web of the girder.
- d. Weld it to the compression flange.

With this method, the fitting of the stiffener will comply with the above AASHTO specs, yet it is not welded to the tension flange, nor is it a problem to insert. An alternate method: -

2. Use a stiffener which is cut short about 1 in. Fit it against the compression flange and weld it to the web of the compression flange. It is not welded to the tension flange. Experience indicates the 1 in. gap at the lower tension flange will present no maintenance problem. Although this does not comply with the above AASHTO requirement, many girders for highway bridges are fitted with stiffeners in this manner.

Plate girder research at Lehigh University has indicated that the stiffener does not have to make contact with the tension flange to develop the ultimate capacity of the girder. They recommended the stiffeners be cut short as described in the alternate method above (2). Dimension of the distance between the lower tension flange and the stiffener is set at four times the web thickness.

There is no clear-cut answer as to whether continuous or intermittent fillet welds should be used to attach the stiffener to the web. This latest research at Illinois on stiffeners indicated that fatigue failures occurred at the terminations of fillet welds, regardless of whether they were continuous or intermittent. Naturally, a continuous weld will have fewer terminations, hence, less areas for potential fatigue cracks.

Where large, intermittent fillet welds are specified, for example, 3/8 in., replacement with 1/4 in. continuous fillet welds made using automatic welding equipment achieves a considerable saving in cost. Where small "possibly 1/4 in." intermittent fillet welds are specified, savings resulting with the introduction of continuous welds and automatic equipment becomes questionable.

In some instances, with thin, deep web plates, a smaller size weld would tend to reduce distortion. In this case, automatic welding would be of benefit, provided this substitution of automatic continuous welds for intermittent welds does not increase weld length to any major extent.

WELDABLE STRUCTURAL STEELS

Weldability of a steel refers to the relative ease of producing a satisfactory joint. A steel is said to be ideally weldable if the required weld joint can be made without difficulty or excessive cost. A properly made welded joint in structural steel has physical properties superior to those of the base metal.

Steel plates and sections as rolled in the steel mill undergo a rather slow rate of cooling after hot rolling. Because of their greater mass, thick sections cool more slowly and also have less reduction during rolling than thinner sections. As a result, the carbon content of the material scheduled for thicker plate as well as the alloy content might have to be increased slightly to maintain the yield strength.

A weld will cool faster on a thick plate than on a thin plate. This fact, combined with the possible higher chemistry, tends to produce welds of higher tensile strength but lower ductility. For this reason, preheating is usually required when welds are to be made on thicker plates.

Table V presents the composition and properties of the various types of ASTM steels which are commonly used in bridges, buildings, and other structures. These steels are readily weldable with both hand and mechanized welding processes.

A-7 steel has been commonly used structural steel for welded bridges and buildings for many years. Although specifications are not intended to control the carbon content, it was felt the upper limit of tensile strength of 75,000 psi would limit this to steels of reasonable carbon content. This material has seen extended use in successfully welded structures, some of which have incorporated plates up to 5 in. thick.

A-373 steel has definite carbon content specifications. Bridge specifications and building codes today are written to permit the use of A-7 steel when its composition conforms to A-373. The manganese content for shapes and plates is constant in this material. Its strength is maintained in thicker plates up to 4 inches, by increasing the carbon content from 0.25% up to 0.27%.

A-36 is a relatively new steel and, like A-373, has carbon content specifications. It has a minimum yield strength of 36,000 psi. The AISC allows a 10% increase in its stress allowables.

The top limit on carbon is 0.27% for 4 in. plates and 0.29% for 8 in. thick plates. It has wide acceptance in both the building and bridge field.

A-441 is a rather new classification. It is a low-alloy weldable steel which previously was classed as A-242. The difficulty with this latter classification is that this group also includes steels containing some alloying additions for increased corrosion resistance. These alloy additions might detract from the steel's weldability. Atmospheric corrosion resistance of A-441 is approximately twice that of structural carbon steel.

It is pretty well agreed that good notch toughness, low transition temperature and good weldability call for a silicon killed steel with a rather low carbon content, fairly high manganese-to-carbon ratio and sufficient alloy content to make up the required minimum yield strength.

A-36 steel has a higher manganese-to-carbon ratio than A-373. It has good weldability and 10% higher strength allowables in the building field. This fact, combined with a recent reduction in base price which makes it competitive to A-7, justifies its consideration for most welded structures. As a result, A-36 steel is rapidly replacing A-7 and A-373 in the building field. It is expected that this steel will also experience rapidly expanding use in bridge construction.